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Risk of Landslides in Shallow Soils and Its Relation to Clearcutting in Southeastern Alaska

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ABSTRACT. A significant increase in the frequency of landslides in shallow soils on hillside slopes of southeastern Alaska following timber harvest by clearcutting has been observed. This phenomenon relates to the loss of root strength and evapotranspiration stress that follows the cutting of the trees. A method for evaluating the landslide risk is described in this paper. A hillside with a nearly uniform slope is represented by an infinite slope and the piezometric level required for shear failure is computed. A one-dimensional infiltration-seepage model is used to calculate the response of the piezometric level to rainfall. Weather data are used to calculate the probability of the piezometric level exceeding the value required for slope failure. Uncertainties in soil strength and slope angle may also be accounted for in the calculation of failure probability. Field data obtained from a site near Hollis, Alaska, are used to illustrate the method of risk evaluation and cost analysis. FOREST SCI. 26:495–510.

ADDITIONAL KEY WORDS. Failure probability, piezometric level, slope stability.

TIMBER HARVEST BY CLEARCUTTING is a common practice in the coniferous forests of the Cordilleran area. Among the many ecological problems caused by clearcutting in this area are the effects on the stability of steep hillside slopes. Data have been presented to show that clearcutting increases the creep rate of slopes (Barr and Swanston 1970, Gray 1970). Empirical evidence gathered in southeastern Alaska (Bishop and Stevens 1964, Swanston 1969) indicates that the frequency of landslides classified as debris avalanches increases significantly a few years after clearcutting. The debris avalanches in southeastern Alaska usually occur during periods of heavy autumn rain and begin with a small slide located near the top of the slope. Because of the high water content, the disturbed soil flows rapidly downhill as a viscous fluid and erodes much of the soil along its path. This results in a scar that extends from the initial slip to the bottom of the slope (Fig. 1). The probable occurrence of debris avalanches and its consequences are clearly important factors to be considered in the planning of logging operations.

Because the various factors that contribute to the development of a debris avalanche cannot be predicted with precision, a probabilistic approach to risk evaluation is appropriate. This approach is comparatively simple when applied to slides on approximately uniform or planar slopes with a shallow soil cover underlain by impervious bedrock such as those near Hollis, Alaska. Hence, solutions for these special conditions are readily obtained. The solutions presented

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FIGURE 1. View of south-facing slope of Maybeso Valley, 1975 (light colored stripes are slide scars).

here also serve to illustrate the general approach of probabilistic risk evaluation and its application to forest management practice. This paper first summarizes the mechanism of the debris avalanche and the environmental factors that control slope stability. It then outlines procedures that may be used to evaluate risk of debris avalanches and assess potential damage or loss. Analysis of the slopes near Hollis is used as an example.

MECHANISM OF LANDSLIDES

Figure 2 shows a hillside with a nearly uniform slope. The depth of the soil cover is $(h + h')$[1] and ab indicates the bedrock surface which is parallel to the ground surface. Such a slope may be analyzed as an infinite slope (e.g., Lambe and Whitman 1969). The soil mass is shown as a block abcd. The forces that act on the mass include the weight of the soil W_s, the weight of the trees W_t, the wind force on the trees F_w, and the shear and normal forces on the surface ab, T and N, respectively. The shearing resistance along ab is defined by

$$S = [c' + (\sigma - u)\tan \phi' + s_r]\ell \tag{1}$$

where c' and ϕ' are the cohesion and angle of internal friction in terms of effective stresses, respectively, $\sigma = N/\ell$ is the normal stress, u is the porewater pressure, and s_r is the contribution of the roots to shear strength. Shear failure along ab occurs if the force T required for equilibrium of the block is equal to the shearing resistance S,

$$S = T \tag{2}$$

or

$$\left[c' + \left\{(W_s + W_t)\frac{\cos \alpha}{\ell} - u\right\}\tan \phi + s_r\right]\ell = (W_s + W_t)\sin \alpha + F_w. \tag{3}$$

The forces W_s, W_t, F_w may be estimated without too much difficulty. The

[1] All symbols are collected and defined at the end of the text.

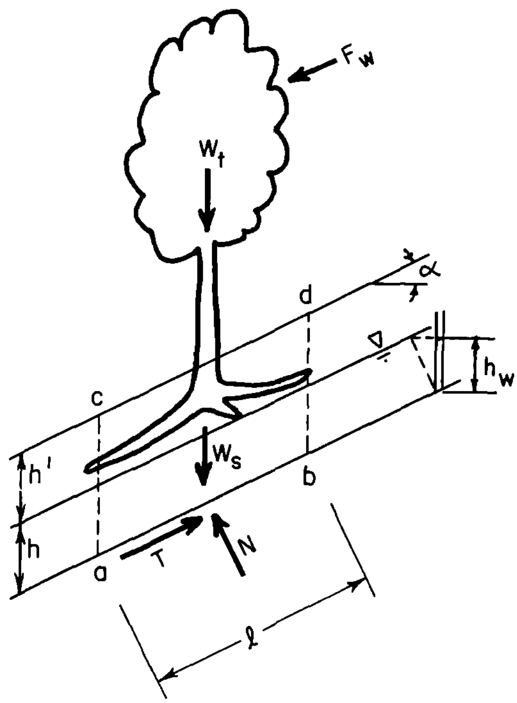

FIGURE 2. Forces on sliding soil mass (symbols are explained in text).

values of c' and ϕ' should be measured by appropriate shear tests and s_r may be determined from root strength and root density. The results of tests on live roots of Sitka spruce (*Picea sitchensis* (Bong.) Carr.), western hemlock (*Tsuga heterophylla* (Raf.) Sarg.), and Alaska yellow-cedar (*Chamaecyparis nootkatensis* (D. Don) Spach) in southeastern Alaska indicate that (Wu and others 1979)

$$s_r = 1.2t_r \tag{4}$$

and t_r is given by

$$t_r = \Sigma T_{ri} n_i \tag{5}$$

where T_{ri} = failure load of a root with diameter i, n_i = number of roots with diameter i per unit area of soil.

The porewater pressure u is related to the elevation of the groundwater table. For seepage parallel to the ground surface (Fig. 2)

$$u = \gamma_w h_w = \gamma_w h \cos^2\alpha \tag{6}$$

where γ_w is the unit weight of water and h_w, h, and α are as shown in Figure 2.

At a specific location, when all the above quantities are known one may calculate the safety factor of a slope, defined as the ratio

$$F = S/T. \tag{7}$$

A safety factor of one or less indicates that failure should occur and a safety factor larger than one indicates stability. Alternatively, the value of h which would cause failure (denoted by h_f) can be calculated given the forces W_s, W_t, and F_w, soil properties, and slope angle.

In the planning of timber harvest the question would be: what is the steepest

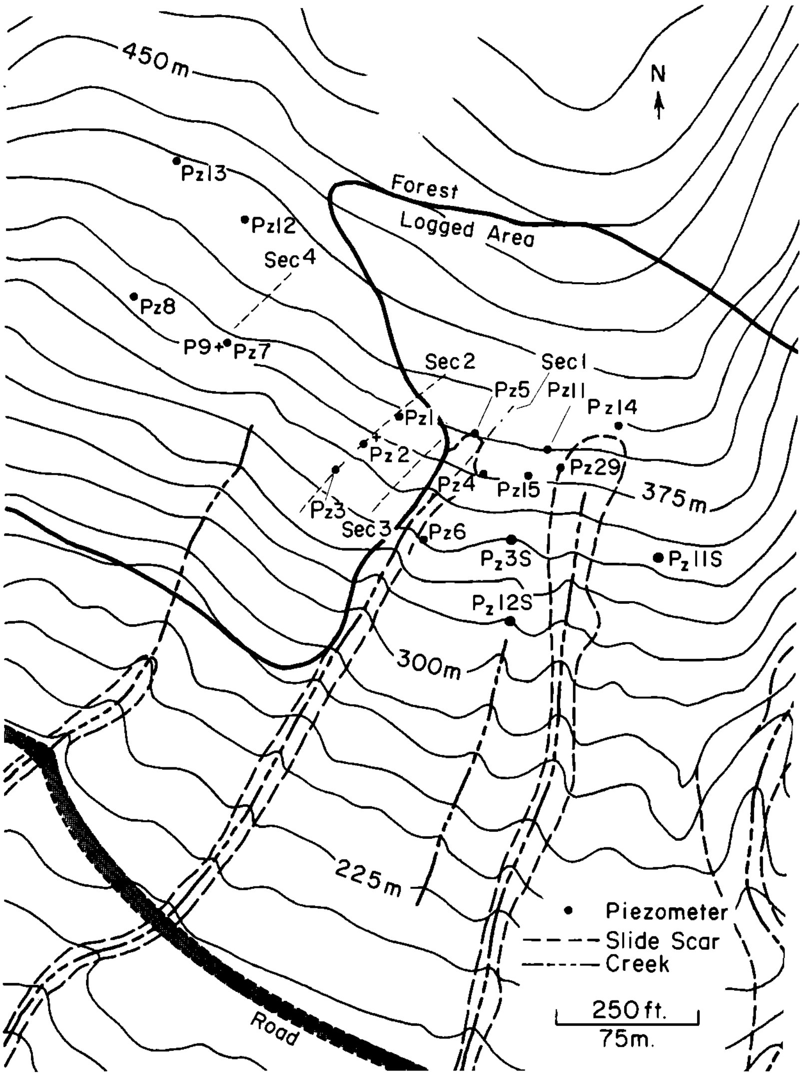

FIGURE 3. Locations of piezometers, Maybeso Valley.

slope (maximum α) that could be logged without causing failure? The value of α required for failure could be calculated from Equation (3) if the forces, soil properties, and piezometric head are known. However, all of these quantities can only be estimated and uncertainties are involved. The piezometric head h_w may be expected to fluctuate with rainfall and other seasonal effects and is difficult to predict. In the probabilistic approach, we estimate the probability that the piezometric head h_w will be equal to or exceed a given value during a specified time interval. For given slope angle α, soil properties, W_s, W_t, and F_w, the probability that h will be equal to or greater than the calculated h_f is also the probability of failure. In addition, if the area is large, the soil properties c' and ϕ' may vary over a considerable range as natural soils are nonhomogeneous. The slope angle α may also be expected to vary over a large area due to local topographic features such as drainage depressions. Estimates of their mean values and their dispersions

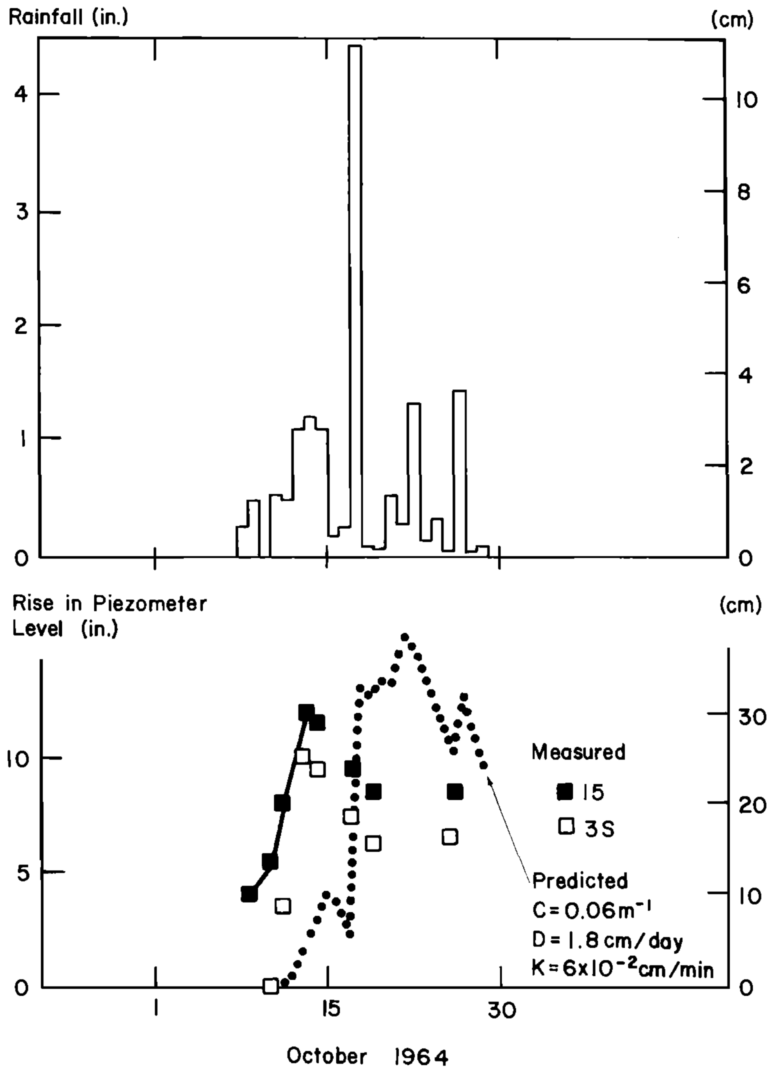

FIGURE 4a. Piezometric levels, 1964, Maybeso Valley.

from the means can be made and the uncertainties can also be accounted for in the probability analysis. The decision problem then becomes one of choosing the acceptable failure probability.

PIEZOMETRIC HEAD

On hillside slopes with shallow soil cover underlain by impervious bedrock such as that shown in Figure 2, the piezometric head usually rises rapidly during rainstorms, followed by a rapid drop. The characteristics of the rise and fall in relation to the rainfall depends on the slope angle, depth of the soil cover, soil permeability K, the slope of the soil moisture-suction curve C, and evapotranspiration loss. Clearcutting may alter these factors by various degrees and in turn may affect the response of the piezometric head to rainfall. This is illustrated by the results of porewater pressure measurements in the Maybeso Valley near Hollis (Swanston 1967, Wu and others 1979).

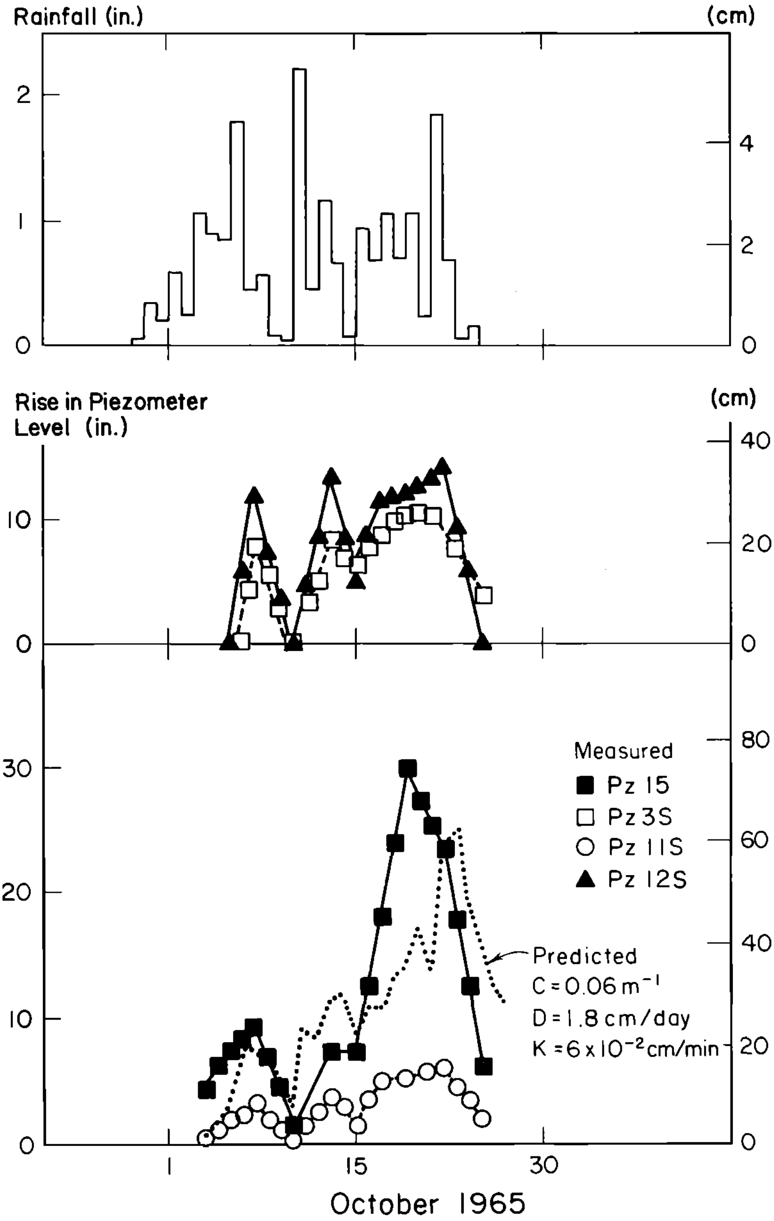

FIGURE 4b. Piezometric levels, 1965, Maybeso Valley.

Figure 3 shows the instrumentation on the slope. The measured porewater pressures in late September and October of 1964 and 1965, about 4 years after a part of the area was logged by clearcutting, are shown in Figures 4a and b. Figure 4c shows the measured porewater pressures in late September and October 1974 after a regrowth of Sitka spruce has been established on the logged slopes. During the relatively dry summer months the piezometric level is very small or zero. Substantial rises in the piezometric level occur only after the autumn rains begin as shown in Figure 4. To compare the piezometric levels under different conditions, consider first the piezometric levels in 1974. The piezometric levels in the

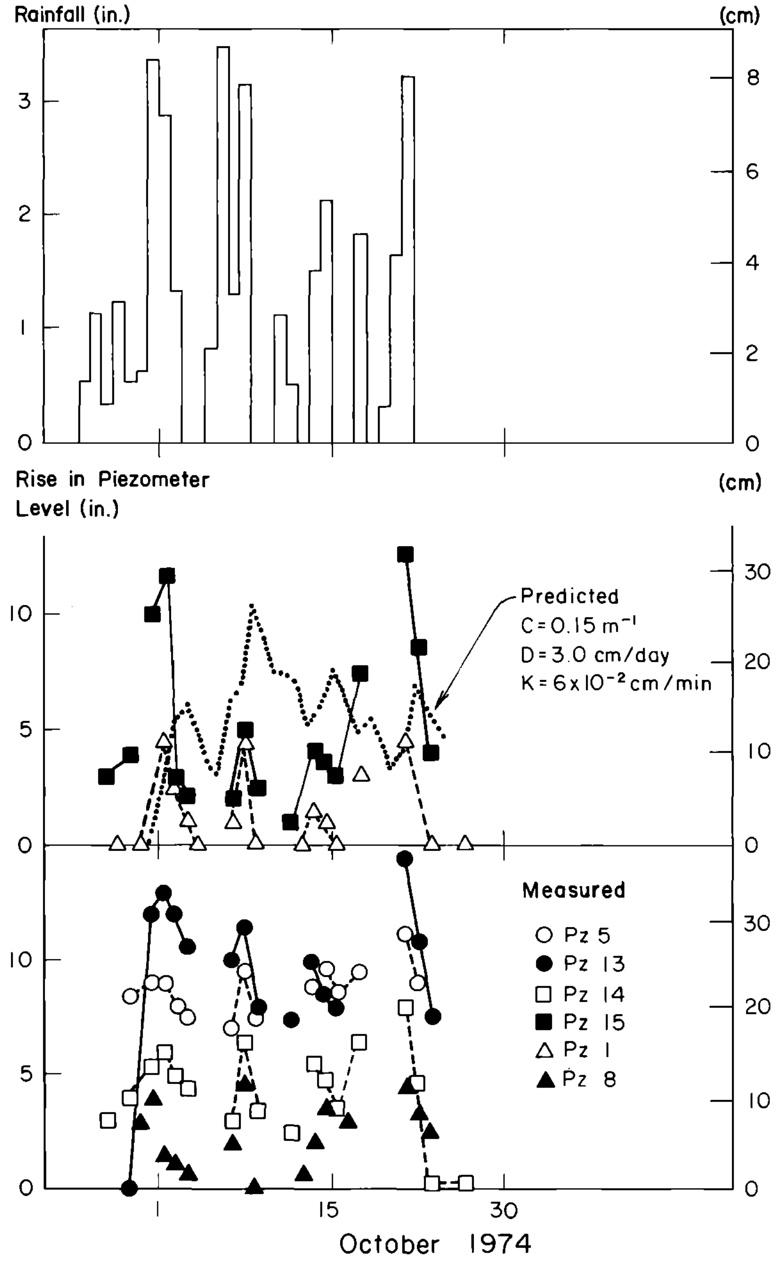

FIGURE 4c. Piezometric levels, 1974, Maybeso Valley.

regrowth area (Pz 14, Pz 15) are about the same as those in the uncut forest (Pz 13). However, comparison of the piezometric levels of 1974 with those of 1965 shows significant differences. In piezometer Pz 15, which was replaced in 1974, the measured level in 1965 is much higher than that in 1974, although the rainfall intensity for 1965 is considerably lower. Comparison between 1965 and 1974 porewater pressures in other piezometers is more difficult because the piezometers are not placed in precisely the same locations and the porewater pressures

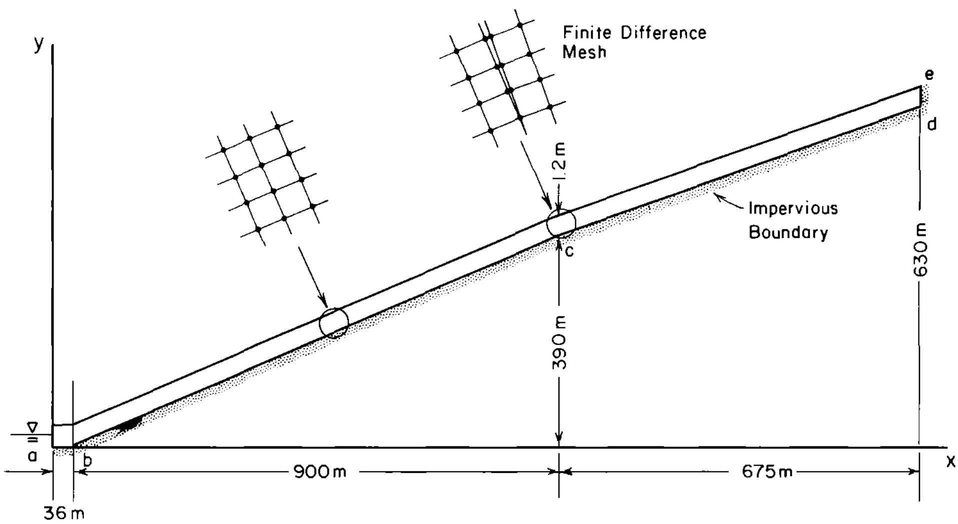

FIGURE 5. Simplified cross section of slope and finite difference mesh used for seepage computation.

appear to vary considerably within short distances. Nevertheless, it can be seen that the porewater pressures in 1965 are at least as large as those in 1974, although the rainfall in 1965 is considerably lower. The record for 1964 is not as reliable because piezometer readings were not made every day as they were in 1965 and 1974. However, the measured piezometer rises and rainfall in 1964 are both close to those of 1965.

The fluctuation in the piezometric levels is the net result of rainfall entering the ground, the evapotranspiration loss, and the drainage loss due to seepage and interception. The rainfall may be obtained from the precipitation record. The drainage loss due to seepage was analyzed by considering the slope as two-dimensional and solving the differential equation for two-dimensional flow (Bear 1972)

$$\frac{\partial}{\partial x}\left[K(\underline{h})\frac{\partial \underline{h}}{\partial x}\right] + \frac{\partial}{\partial z}\left[K(\underline{h})\frac{\partial \underline{h}}{\partial z}\right] = (1 - n)G_s C(h_w)\frac{\partial \underline{h}}{\partial t} + q \qquad (8)$$

where $\underline{h}$ = the head, $\frac{\partial \underline{h}}{\partial x}$ and $\frac{\partial \underline{h}}{\partial z}$ = gradients in the x and z directions, t = time, K = the permeability, $C(h_w)$ = slope of soil moisture-suction curve, q = source term, n = porosity, and G_s = specific gravity. Equation (8) may be solved numerically by the method of finite differences (Carnahan and others 1969). The Maybeso valley slope is simplified as shown in Figure 5 together with the finite difference grid. With no rainfall, the computed drop in water table due to seepage is about 0.6 cm/day.

The potential evapotranspiration has been estimated by Thornthwaite's method to be of the order of 90 cm/year (Gass and others).[2] While the distribution of evapotranspiration throughout the year is not known for this location, results of measurements by Fritschen and others (1977) indicate that evapotranspiration of a Douglas-fir (*Pseudotsuga menziesii* (Mirb.) Franco) in Washington continued

[2] Gass, C. R., R. F. Billings, and M. E. Stephens. Soil Management Report for the Hollis Area, South Tongass National Forest, Ketchikan, AK.

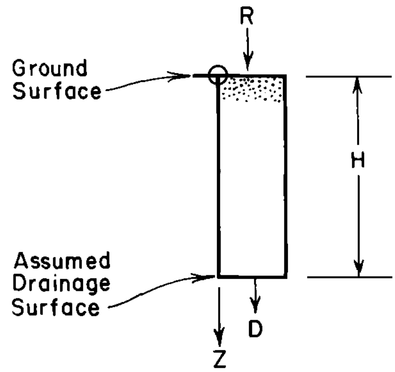

FIGURE 6. One-dimensional flow (symbols are explained in text).

at a nearly uniform rate well into November. Since the climate at Hollis is similar one may infer that evapotranspiration continues during the autumn rain season. Assuming that all the potential evapotranspiration occurs during the 4 months from July through October gives an evapotranspiration rate of 0.75 cm/day as an upper limit. It is known that evapotranspiration can vary over a wide range depending on solar radiation, air temperature and humidity, wind speed, and soil moisture (Molchanov 1960, Penman 1967, Sellers 1969). While available data are inadequate for accurate representation of the evapotranspiration, the rough estimate indicates that it is of the same order of magnitude as the seepage loss and hence both would strongly influence the piezometric level during and after rainstorms.

In order to analyze the fluctuations in the piezometric level and determine the effect of evapotranspiration and other factors we simplify the problem and consider only one-dimensional infiltration (Fig. 6). Here H represents the thickness of the previous soil lying on top of an impervious base. Equation (8) then reduces to

$$K\frac{\partial^2 \underline{h}}{\partial z^2} = (1 - n)G_s C(h_w)\frac{\partial \underline{h}}{\partial t} + q. \tag{9}$$

The boundary conditions are

$$z = 0, \quad K\frac{\partial h}{\partial z} \begin{array}{ll} = R & \text{if } \underline{S} < 1 \\ = 0 & \text{if } \underline{S} = 1 \end{array} \tag{10}$$

where R is the rainfall and $\underline{S}$ is the degree of saturation. The evapotranspiration and seepage parallel to the slope are combined into a discharge rate D. It is assumed that this occurs at the bottom of the previous soil layer whose thickness is H, or

$$z = H, \quad K\frac{\partial \underline{h}}{\partial z} = D. \tag{11}$$

The soil properties are K, n, G_s, and C, and the input parameters are R and D. If all these are known, Equations (9), (10), and (11) may be used to compute the values of $\underline{h}$ at different depths and times. The groundwater surface at any time is the point at which $h_w = 0$. This allows us to obtain the fluctuations of h with time. However, in the present case we wish to evaluate the effect of D on measured piezometric level. This was accomplished empirically by calibrating it against observed fluctuations of the groundwater level. Calculations were made with different values of D to obtain the best fits to the data. These are shown as

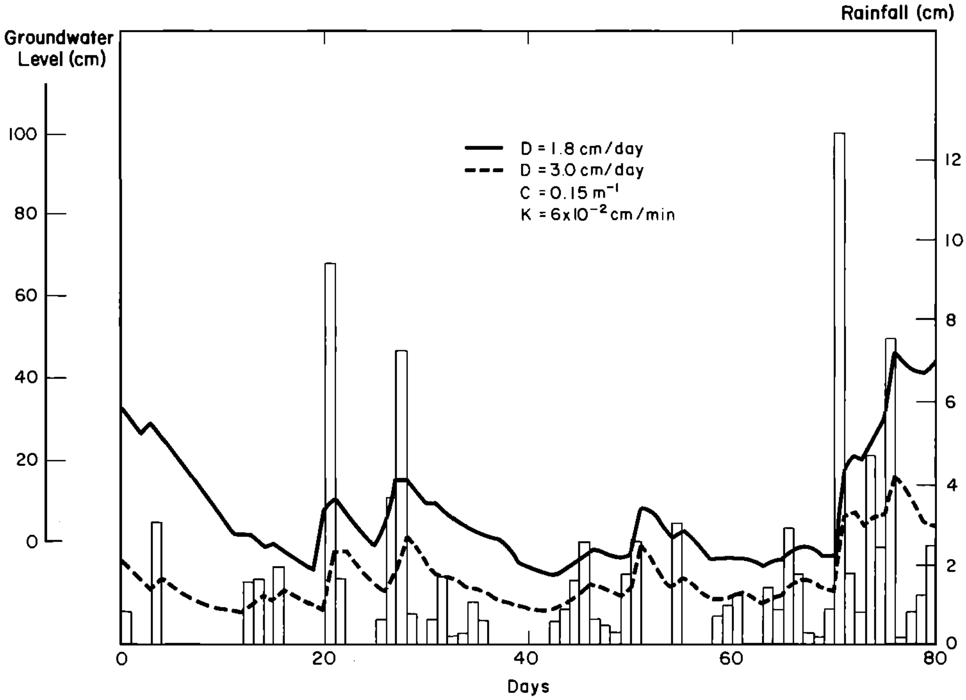

FIGURE 7. Simulated piezometric levels.

dotted lines in Figures 4a, 4b, and 4c and the values of D and C are also indicated. We note that the piezometric levels measured in the different piezometers do not always show the same pattern of fluctuation. The computed curves are taken as representative of the average conditions in the upper part of the slope (Pz 15 in Figs. 4a and b; and Pz 5, Pz 13, Pz 14, Pz 15 in Fig. 4c). Comparing the values of D and C used in the analytical models in Figures 4a, 4b, and 4c, we see that the difference in porewater pressure response between 1965 and 1974 represents a change of D from 1.8 cm/day in 1965 to 3.0 cm/day in 1974 and of C from 0.06 to 0.15 m^{-1}. These values of D are of the same order of magnitude as the sum of the estimated seepage loss and evapotranspiration loss. The increased value of D for 1974 may be attributed to increased evapotranspiration loss because of the regrowth. It can also include the effect of a lower antecedent soil moisture because of the regrowth. This may also be the reason for the larger value of C in 1974 since C is known to increase with decreasing soil moisture.

While the one-dimensional model is approximate and does not account for all of the factors that contribute to fluctuations of the piezometric level, the preceding example shows that if adequate data are available for a site, an analytical model may be calibrated with the data and used to simulate the average groundwater level. The actual groundwater level at a particular point may be expected to differ from the predicted average as shown in Figure 4. This difference is the error introduced by uncertainties in the model and the parameters.

RISK EVALUATION

The results cited in the preceding sections indicate that the various parameters that affect slope stability cannot be determined with precision even with extensive field instrumentation. A rational method that takes into account the uncertainties

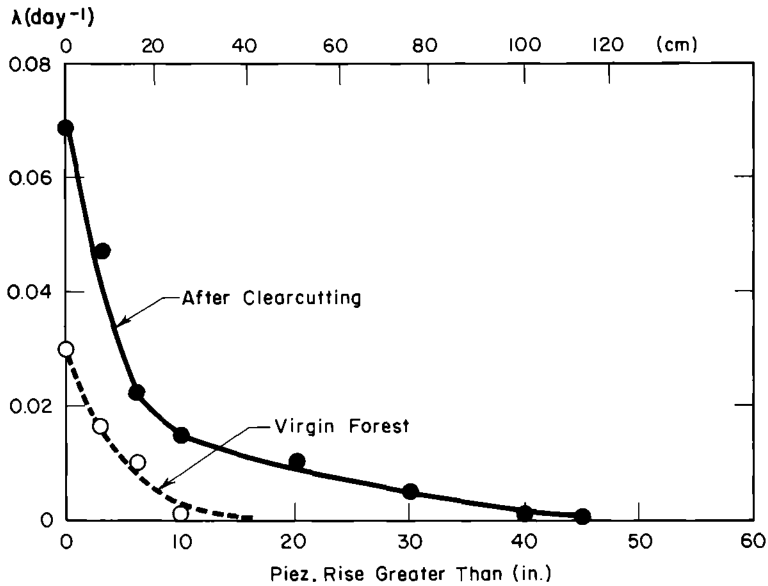

FIGURE 8. Distribution of piezometric levels.

is the use of probability theory to assess risk. As an illustration, this approach is used to assess the risk of debris avalanches in the Maybeso Valley. We first simplify the problem by assuming that all the parameters except the porewater pressure are precisely known. Then the problem consists of estimating the probability that u will reach a level that will satisfy Equation (3).

In order to compute porewater pressure, it is necessary first to obtain weather data for the period under consideration. We assume that the nature of precipitation in the future will remain the same as in the past. Then the rainfall record may be generated by Monte Carlo simulation. We first simulate the daily occurrence of rainfall and no rainfall as a Markov chain (Haan and others 1976) in which the states 1 and 2 represent rain and no rain, respectively. The transition matrix is

$$P = \begin{bmatrix} P_{11} & P_{12} \\ P_{21} & P_{22} \end{bmatrix} \tag{12}$$

in which P_{ij} represents the transition from state i to state j. Because heavy rainstorms in the Hollis area occur mostly during the months of September, October, and November, the recorded rainfall at Ketchikan for these months from 1972 through 1974 (Environmental Science Services Administration 1972–74) were used to obtain

$$P = \begin{bmatrix} 0.78 & 0.22 \\ 0.34 & 0.66 \end{bmatrix}. \tag{13}$$

The weather for the first day was generated using the fractions of days with and without rain. The presence or absence of rainfall on the subsequent days was generated with the transition matrix, Equation (13).

For a rainy day, the amount of rainfall was generated from the distribution of rainfall for the months of September, October, and November 1972–74. The one-dimensional model, Equation (9), was used to compute the groundwater level. An example of simulated rainfall and groundwater level is shown in Figure 7. To

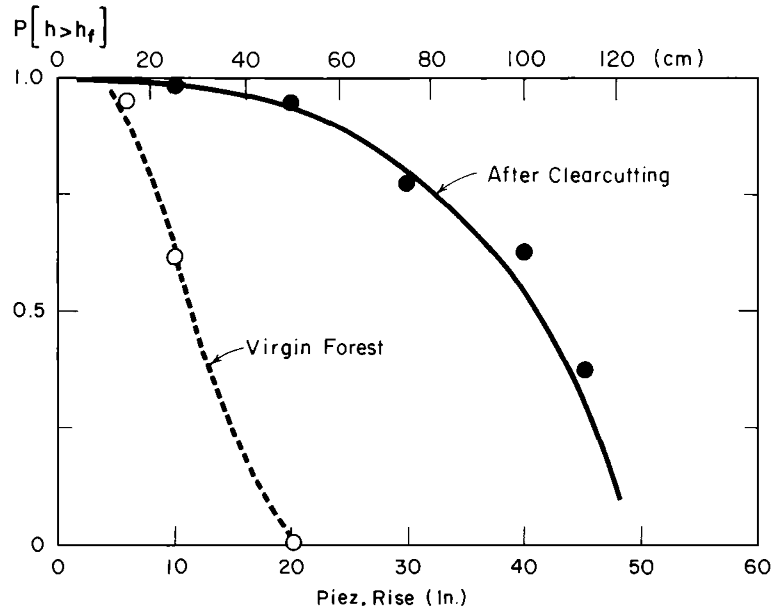

FIGURE 9. Probability of piezometric height (h) exceeding the height required for failure (h_f).

determine the probability of the groundwater level h exceeding a given value at least once during the annual rain season of approximately 3 months, we should carry out the simulation for a large number of 3-month periods. Then the probability is the number of rain seasons in which the level is exceeded divided by the total number of rain seasons simulated.

Alternatively a simplified approach may be taken. We determine λ, the number of times a given height h is exceeded per unit time interval from a simulated record such as Figure 7. Figure 8 shows the relation between λ and h. If we assume that the occurrences of peak groundwater level are independent of each other, then the probability that a given groundwater level h will occur at least once during a time interval t is given by the Poisson distribution

$$P(h \geqslant h_1) = 1 - e^{-\lambda t} \tag{14}$$

where h_1 = given value of h. For failure to occur, we should have $h = h_f$. Hence, the probability of failure is

$$P_f = P\ (h \geqslant h_f) = 1 - e^{-\lambda t}. \tag{15}$$

To calculate the failure probability, we consider t to be the period from 4 to 8 years after clearcutting, because, for the species in the Hollis area, the roots die after the tree is cut. Decay of roots progresses so that $S_R \rightarrow 0$ after about 4 years (Wu and others 1979). Significant regrowth is established about 8 years after logging. The rain season of September through November for 4 years constitutes about 300 days. The values calculated by Equation (15) are shown in Figure 9. Computation of h_f was made with Equations (3) and (5); the average soil properties $c' = 5.3$ kPa, $\phi' = 34.7°$; $s_r = 0$, $\alpha = 39°$, $h + h' = 1.22$ m, $W_t = 0$, $F_w = 0$, which represent the slope after clearcutting. We obtained $h_f = 0.84$ m. Hence, the probability of failure is

$$P_f = P(h \geqslant 0.84 \text{ m}). \tag{16}$$

From Figure 9, we see that the probability of this occurrence is about 0.75.

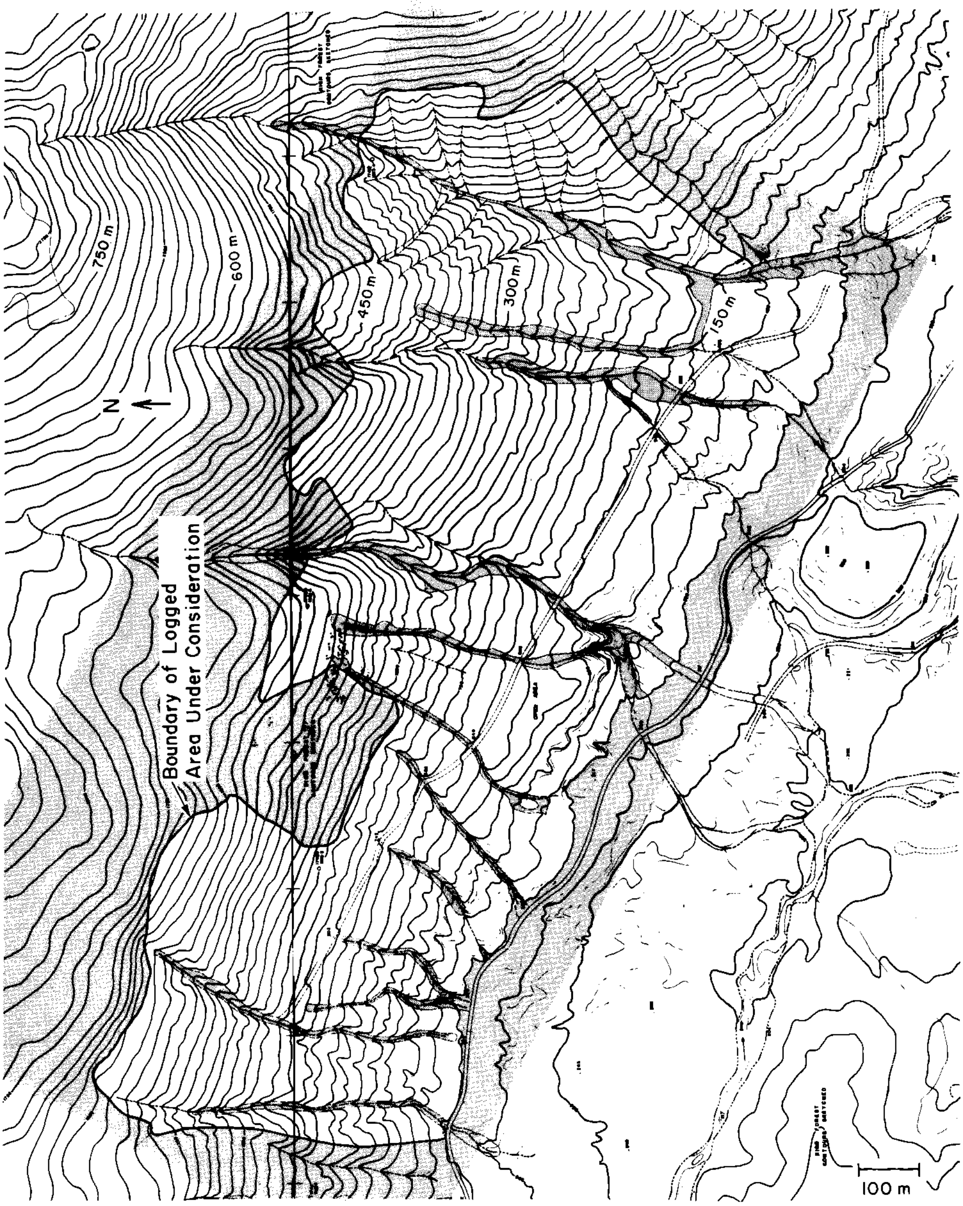

FIGURE 10. Map of a part of logged area, Maybeso Valley, showing landslide scars.

If one chooses an acceptable failure probability of, say, 0.30, then the slope angle α which will have $P_f \leq 0.30$ can be calculated. This would indicate the steepest slope that could be subjected to clearcutting without incurring a risk greater than 0.30. From Figure 9, we see that the probability of having a rise in groundwater level of 1.14 m is about 0.30. The slope that would be on the verge of failure when $h = 1.14$ m would have $\alpha = 34°$. Hence, slopes steeper than this should not be logged if the failure probability is to be kept below 0.30. If a failure probability of 0.10 is required, similar calculations give $\alpha = 32°$ as the steepest slope that can be logged.

The failure probability calculated above does not account for uncertainties about the soil strength and slope geometry and the model uncertainty. These uncertainties and their effects can be evaluated as shown in the following example. If the shear strength s, slope angle α, and the model uncertainty on h, N_h, are considered as random variables, then failure represents the joint occurrence of s, α, N_h, and $(h \geqslant h_f)$, or

$$P_f = P[(h \geqslant h_f) \cap (s, \alpha, N_h)]. \tag{17}$$

We assume that s, α, and N_h are independent but h_f is a function of s and α. Hence,

$$P_f = \sum_{s,\alpha,N_h} P[s]P[\alpha]P[N_h]P[(h \geqslant h_f)\,|\,(\alpha, s, N_h)] \tag{18}$$

where $P[(h \geqslant h_r)\,|\,(\alpha, s, N_h)]$ is given in Equation (14) and Figure 9. As an example we use the data from the Hollis site which show that the minimum and maximum shear strengths are respectively $c' = 2.1$ kPa, $\phi' = 37°$, and $c' = 6.0$ kPa, $\phi' = 44°$ (Wu and others 1979). We assume that the distribution of shear strength is uniform within these limits and there is no uncertainty about α and the model and obtain $P_f = 0.70$.

Expected Cost

The preceding examples serve only to illustrate the method of computation and do not constitute recommendations on acceptable risk. The choice of the acceptable risk or failure probability is properly a management decision. Logically, such decisions would be based on estimated socioeconomic values. Statistical decision theory may be used to estimate the cost due to probable failures. The measure used is the expected cost, defined as (Benjamin and Cornell 1970)

$$E = P_f C_f \tag{19}$$

where P_f is as defined by Equation (18) and C_f is the cost of a failure. This allows one to compare the costs with monetary benefits derived from logging. The cost of landslides is one of the items that should be included in the cost estimates.

If the volume of soil eroded is considered to be the cost, we may use the experience in the Maybeso Valley to illustrate the loss caused by debris avalanches. The map in Figure 10 shows a portion of the north slope of the Maybeso Valley. The heavy line in the figure indicates the logged area under consideration. The shaded areas indicate slide scars with no vegetation. These have a total area of approximately 0.17×10^6 m^2. The area affected by slides is about 9 percent of the area logged. An estimate of the volume of soil removed may be made with the approximation that the depth of the failure surface is 1 m below the original ground surface. The calculated volume of soil removed is then 0.17×10^6 m^3, or about 0.1 m^3/m^2 of logged area. A large area as the one under consideration may be considered to be composed of many small potential slides, each one of which represents an independent occurrence. Then the soil loss of 0.1 m^3/m^2 is equal to the expected cost E. For $P_f = 0.70$, $E = 0.1$ m^3/m^2; Equation (19) gives $C_f = 0.14$ m^3/m^2. If clearcutting were restricted to slopes with α less than 39°, P_f would be less than 0.70. The new expected cost can be computed with Equation (19) using the new P_f and $C_f = 0.14$ m^3/m^2.

Concluding Remarks

The procedure outlined in this paper may be used to assess the risk and cost of landslides due to clearcutting. It is based on the principles of soil mechanics and seepage. Soil properties needed for the analysis may be determined from standard tests. Uncertainties caused by natural variations in soil properties, slope angles, and precipitation, and inaccuracies in the analytical models may be accounted for.

The prediction of the porewater pressure changes by the theory of infiltration and seepage requires information on evapotranspiration which is often not available. Hence, calibration of the theoretical model with measured porewater pressure is needed to overcome this problem and evaluate the model uncertainty. It will be necessary to do this for various climates, vegetation covers, and subsurface conditions.

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Symbols Used in Text

C = slope of soil moisture-suction curve
C_f = cost of a failure
c' = cohesion
D = water discharge rate
E = expected cost
F = safety factor
F_w = wind force
H = thickness of soil layer above groundwater level
h' = soil depth
h = soil depth below groundwater level
h_w = piezometric head
$\underline{h}$ = total head
h_f = piezometric head required to cause failure
K = soil permeability
ℓ = length of slip surface
N = normal force
N_h = model uncertainty factor
n = soil porosity

n_i = number of roots with diameter i per unit area of soil
P = probability
P_f = probability of failure
Pz = piezometer
q = source term
R = rainfall
S = shear resistance (force)
$\underset{\sim}{S}$ = degree of saturation
s_r = contribution of roots to shear strength
s = shear strength (stress)
T = shear force
T_{ri} = failure load of a root with diameter i
t = time
u = pore water pressure
W_s = weight of soil
W_t = weight of trees
α = slope angle
γ_w = unit weight of water
λ = the number of times a given h is exceeded per unit time interval
σ = normal stress
ϕ' = angle of internal friction